A METHOD FOR PUSHOVER ANALYSIS IN SEISMIC ASSESSMENT OF MASONRY BUILDINGS

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SUMMARY

A method for the nonlinear static analysis of masonry buildings is presented, suitable for seismic assessment procedures based on pushover analyses. The method is based on an equivalent frame idealization of the structure, and on simplified constitutive laws for the structural elements. Applications on up to five storey structures are discussed, pointing out some issues regarding modeling hypotheses and calculated response. A possible use of the method in seismic assessment is presented. The procedure makes use of displacement response spectra and of the substitute-structure approach which has been proposed by other authors for reinforced concrete structures. A simple example of the assessment procedure on a two-storey masonry structure is presented. Open questions and future developments are pointed out.

INTRODUCTION

The role of non-linear equivalent static (pushover) analyses is being more and more recognized as a practical tool for the evaluation of the seismic response of structures. Pushover analyses are therefore increasingly being considered within modern seismic codes, both for design of new structures and for assessment of existing ones.

Considering the problem of seismic assessment of masonry buildings, the need for non-linear analysis had been recognized in Italy since the late Seventies. In 1978 and 1981, recommendations on seismic assessment, repair and strengthening of masonry buildings were issued, suggesting the use of an equivalent static, simplified non-linear assessment method which had been proposed and developed in Slovenia by Tomaževic [1978]. Such method, which has undergone several refinements in the subsequent years [Tomaževic, 1997], is based on the so-called "storey-mechanism" approach, which basically consists in a separate non-linear interstorey shear-displacement analysis for each storey, where each masonry pier is characterized by an idealized non-linear shear-displacement curve (typically elastic-perfectly plastic with limited ductility). The conceptual simplicity of the "storey-mechanism" method and its adoption by the Italian recommendations were fundamental in its diffusion among professionals, and the method has been extensively used in Italy since its first introduction in code provisions. However, the simplicity of the "storey mechanism" approach, is paid with a series of limitations which may restrict its application only to some classes of buildings [Magenes and Della Fontana, 1998]. The need for more general methods of analysis has stimulated in Italy the research on the subject, and analytical methods have made significant progress in the last decades, particularly in the field of finite element analyses. However, refined nonlinear finite element modeling does not constitute yet a suitable tool for the analysis of whole buildings in the engineering practice. For this reason, several methods based on macro-element discretization have been developed, requiring a low to moderate computational burden. Within this context, it was felt by the author that several basic ideas of the "storey-mechanism" approach could be used and extended to a broader range of validity, maintaining concepts and idealizations that are familiar to the engineer and obtaining results that can be compared with those of more sophisticated analysis. Following this idea, a simplified method based on an equivalent frame idealization of multistorey walls was developed and implemented at the University of Pavia. This paper describes the model and its possible use in assessment procedures.

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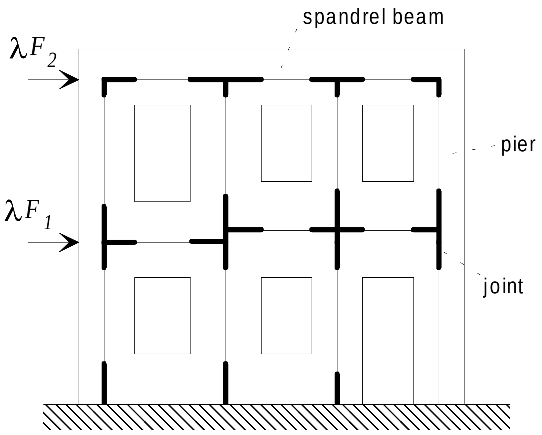

Figure 1. Equivalent frame idealization of a masonry wall.

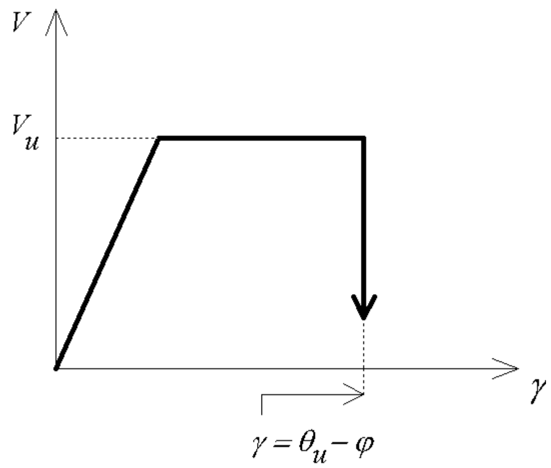

Figure 2. Idealized nonlinear behaviour of a pier element failing in shear.

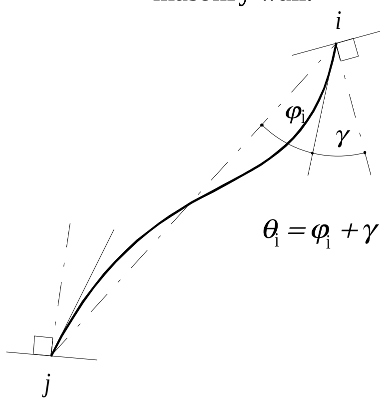

Figure 3. Chord rotation in a beam-column element.

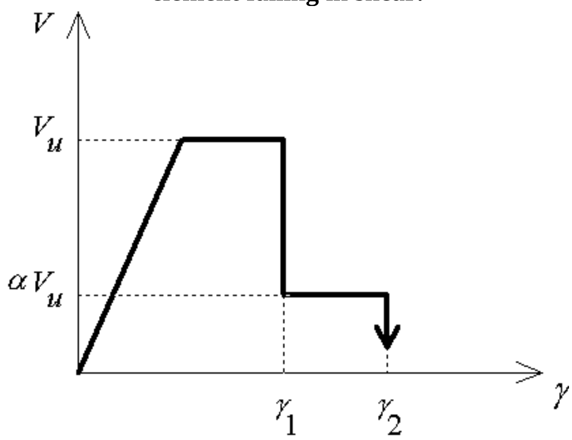

Figure 4. Idealized nonlinear behaviour of a spandrel element failing in shear.

A METHOD FOR THE NONLINEAR STATIC ANALYSIS OF MASONRY BUILDINGS

The model herein described (acronym: SAM for Simplified Analysis of Masonry buildings) was conceived for the global analysis of new and existing masonry buildings, in which the resisting mechanism is governed by in-plane response of walls. Collapse mechanisms due to dynamic out-of-plane response are not considered in the model, and should be evaluated with separate modeling. The global seismic analysis of an unreinforced masonry building is meaningful if proper means, such as ties and/or ring beams, prevent local and global out-of-plane collapses, which otherwise would occur prematurely at low seismic intensities. The model was developed first for plane structures [Magenes and Della Fontana, 1998], and subsequently extended to three-dimensional buildings [Magenes, 1999].

Considering a multistorey masonry wall loaded in plane by horizontal forces, if the geometry of the openings is sufficiently regular, it is possible to idealize the wall as an equivalent frame made by pier elements, spandrel beam elements, and joint elements (Figure 1). The pier element and the spandrel element are modeled as beam-column elements with shear deformation, while the joint elements are supposed infinitely resistant and stiff, and are modeled by means of rigid offsets at the ends of the pier and spandrel elements.

The pier element is supposed to have an elasto-plastic behaviour with limited deformation. The element displays a linear elastic behaviour until one of the possible failure criteria is met. The elasto-plastic idealization approximates the experimental resistance envelope under cyclic actions. The following failure mechanisms are foreseen.

Flexural or "rocking" failure occurs when the moment *M* at any of the end sections of the effective pier length attains the ultimate moment M_u which is a function of axial force, geometry of the section and masonry compression strength f_u. A plastic hinge is then introduced in the section where M_u is attained.

Diagonal shear cracking is defined by the lowest between the strength associated to mortar joint failure and brick unit failure, according to what proposed in [Magenes and Calvi, 1997]. When the failure criterion is met, plastic shear deformation occurs as in Figure 2, where a limit θ_u to the maximum chord rotation is set, beyond which the strength is zeroed. Chord rotation is expressed as the sum of the flexural deformation and of shear deformation $\theta = \varphi + \gamma$ (Figure 3), and is a generalization of the concept of drift for non-symmetric boundary conditions of a pier subjected to flexure and shear. A suggested limit for unreinforced masonry is θ_u = 0.5 %.

Shear sliding can occur in any of the end sections of the pier, and is a function of bedjoint shear strength and of the extent of flexural cracking in the section. Anelastic deformation due to shear sliding is modeled similarly to the case of diagonal shear cracking.

The complete expressions for the strength criteria can be found in [Magenes and Calvi, 1997] and [Magenes and Della Fontana, 1998]. The failure criteria are such that flexural strength is non-zero only in presence of axial compression. No axial tension is allowed, i.e. the axial stiffness of the pier is zeroed for tensile axial deformation.

The spandrel beam element is formulated similarly to the pier element, taking into account the different orientation of bedjoints with respect to the axial force. The possible failure mechanisms are flexure and shear. For flexural failure the formulation is identical to the pier element. For shear strength it is considered that, because of the openings above and below the spandrel element, the bedjoints have almost zero normal stress, and shear strength is therefore provided by cohesion only. The nonlinear behaviour of spandrels failing in shear is depicted in figure 4, in which strength degradation is foreseen for increasing values of shear deformation. By means of the parameters α, γ_1, γ_2 it is possible to obtain a variety of behaviours, from elastic-brittle to elastic-perfectly plastic. This more articulated constitutive hypothesis allows to take into account the tendency to a more brittle post-peak behaviour of spandrels, as compared to piers, which has some relevance on the results.

To analyze three-dimensional buildings, the plane model was extended [Magenes, 1999] by formulating the constitutive laws of piers and spandrels in three dimensions, assuming an independent behaviour of the pier or spandrel element in the two principal orthogonal planes parallel to the element axis. The out-of-plane behaviour is modeled similarly to the in-plane behaviour. Composite walls (i.e. flanged walls or orthogonal intersecting walls) are decomposed in simple walls with rectangular cross section. If the intersecting walls are effectively bonded, it is possible to simulate the bond defining appropriate rigid offsets and imposing the continuity of displacements at the ends of rigid offsets at the floor levels.

An important issue was considered the possibility of modeling the presence of r.c. ring beams, whose role can influence to a large extent the coupling between piers. Ring beams are modeled as elasto-plastic frame elements, which can fail in flexure with plastic hinging. Steel ties can be modeled as elasto-plastic truss elements. Rigid floor diaphragms can be simulated imposing a kinematic constraint among the nodes at the floor level.

VERIFICATION OF THE METHOD

The first applications of the method [Magenes and Della Fontana, 1998] were made on two- and three-storey walls, comparing the results to those obtained by refined plane-stress non-linear finite element analyses with a specific constitutive law for unreinforced brick masonry [Gambarotta and Lagomarsino, 1997]. In such analyses (an example is given in Figure 5) a very good agreement of the results of the two methods was found in terms of overall strength and failure mechanisms, provided that in the SAM method an elastic-brittle behaviour of the spandrels failing in shear was assumed. Although such assumption is conservative and more consistent with the finite element simulations, there is little experimental information on the post-peak behaviour of unreinforced spandrel beams subjected to cyclic actions, so that the question on what kind of modeling hypothesis is more realistic still calls for clear experimental references. Although this modeling issue is not crucial for one- or two-storey buildings, it can have a strong influence on the results for buildings with more than two storeys.

Further analyses on a five-storey wall were made, to evaluate the influence of several modeling hypoteses concerning the strength and stiffness of coupling elements (r.c. beams and masonry spandrels). The five storey wall (Figure 6), taken from an existing building in the city of Catania (built circa 1952), was made of brick masonry, with continuous r.c. beams at each floor. Such a wall was subjected to a “code” pattern of seismic forces gradually increasing proportionally to a scalar, using different assumptions regarding the coupling elements, as described in Table 1. To handle possible softening of the structure before global collapse was reached, the analyses were carried out controlling the displacement of a single point of an external statically determined system which distributed the seismic forces to the floors keeping the desired ratio among the forces.

The calculated global strengths (maximum base shear V_{max}) in the different analyses are summarized in Table 1, and the complete force-displacement curves are reported in Figure 7. The variation in strength is quite significant, showing that the influence of the coupling elements can affect the strength of a multistorey wall by as much as 50% to 100%. At the same time, the global failure mechanism of the wall can vary from a storey mechanism (at the ground floor or at the last floor) to a global overturning of cantilever walls (in case G, where no r.c. ring beam is present), as reflected by the displacement profiles in Figure 8. Such a variety of results shows how the role of the coupling elements should not be overlooked in a seismic analysis.

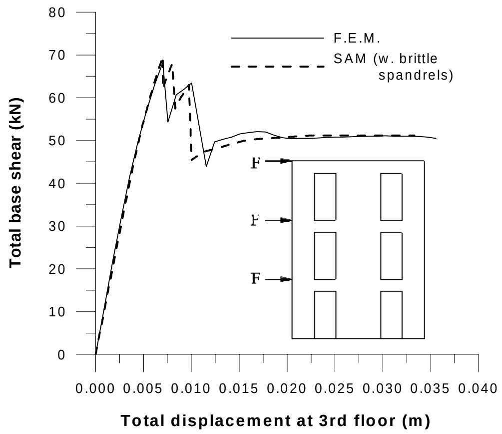

Figure 5. Pushover analysis of a three-storey wall with weak spandrels.

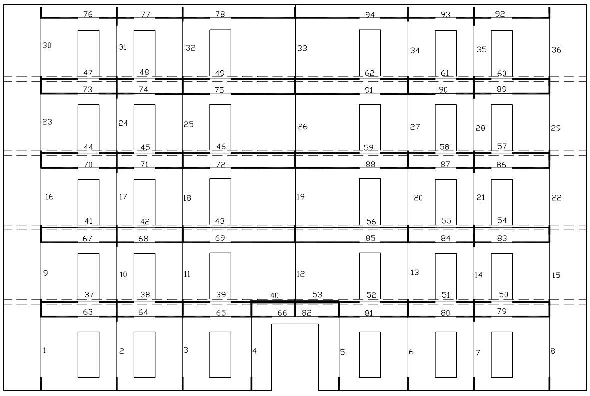

Figure 6. Equivalent frame model of a five storey wall.

Table 1. Summary of the analyses carried out an the five storey wall.

ANALYSIS	HYPOTHESES	V_{max} [kN]	V_{max}/ W_{tot}
A	elastic r.c. ring beam, stiffness calculated according to the uncracked section	1227	0.369
B	elastic r.c. ring beam, cracked section stiffness (1/5 of A)	848	0.255
C	elasto-plastic ring beam; flexural strength calculated according to the probable existing reinforcement	674	0.203
G	only masonry spandrels with no r.c. ring beam	656	0.197
I	coupling elements with no flexural stiffness (cantilever wall system)	477	0.143

V_{max}= maximum base shear; W_{tot} = total weight of the wall.

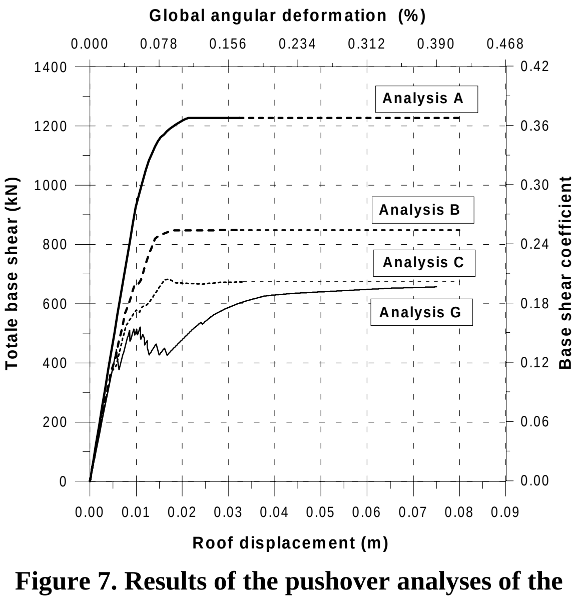

Figure 7. Results of the pushover analyses of the five-storey wall.

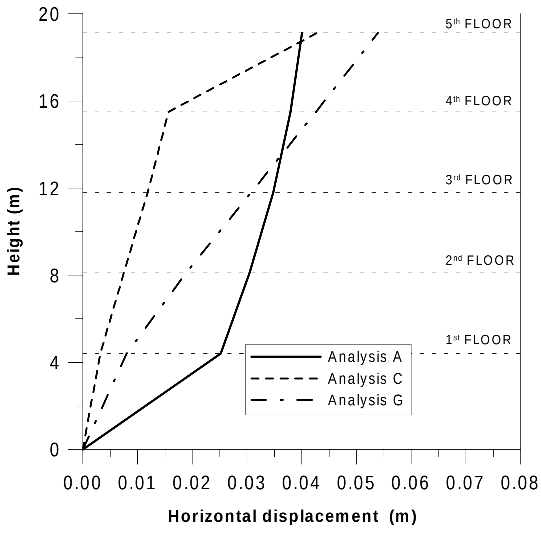

Figure 8. Displacement profiles associated to different collapse mechanisms.

The SAM method was also applied to perform a three-dimensional analysis of the considered five-storey building. The model (approximately 15 x 11 m in plan, 19 m in height) consisted of 390 elements and 195 nodes, for a total of 432 degrees of freedom (assuming in-plane rigidity of floors). As it can be seen, such a model can easily be handeled by any modern personal computer. The calculated strength in the weakest direction with the most realistic hypotheses was V_{max} / W_{tot} = 0.15, which revealed a very high seismic vulnerability. The result is presently being compared with the results obtained by other researchers with different analytical models.

A PROCEDURE FOR SEISMIC ASSESSMENT

The possible use of the proposed model within a simplified seismic assessment procedure is here outlined. It is assumed that the seismic input is given by means of elastic design displacement/acceleration spectra. The procedure proposed herein is based on the use of displacement spectra and on the "substitute-structure" concept [Shibata and Sozen, 1976], which has been adopted in recent proposals of displacement-based design and assessment [Priestley and Calvi, 1997] and which had been outlined for masonry by Magenes and Calvi [1997] in the case of single d.o.f. systems. Other approaches could be envisaged, based for instance on force reduction factor and acceleration spectra, or based on composite displacement-acceleration spectra, and they will be considered for future developments.

The goal of the procedure is to evaluate the deflected shape of the building at peak response. As a start, in this context it will be assumed that the structure is sufficiently regular so that multiple-mode response need not be considered. The main steps of the procedure can be described as follows.

1) Assume a deflected shape $\{\delta^{(0)}\}$, and define a distribution of equivalent static inertia forces $\{F\}$ as:

$$F_i = F_{base} \cdot \gamma_i, \text{ where } \quad \gamma_i = \frac{M_i \delta_i^{(0)}}{\sum M_i \delta_i^{(0)}} \qquad (1)$$

where M_i and $\delta_i^{(0)}$ are respectively the lumped mass and the horizontal displacement at the *i*th degree of freedom, and F_{base} is the total base shear. A possible first choice for $\{\delta^{(0)}\}$ could be obtained by the first mode shape associated to the initial elastic stiffness of the building, or more simply by a set of displacements linearly increasing with height.

2) Perform a nonlinear static pushover analysis up to collapse of the structure under the given distribution of static forces, maintaning the ratios determined by the coefficients γ_i . Collapse may be defined as the attainment of the ultimate drift for individual piers. To handle possible softening of the structure before the attainment of the ultimate limit state, it may be necessary to perform the static analysis in displacement control, as made in the examples described in the previous section, to assure that the desired ratios among the seismic forces are kept.

3) Define an equivalent s.d.o.f. system, with the following characteristics:

$$M_{eq} = \sum_i M_i = M_{tot} \;; \quad \delta_{eq} = \sum_i \gamma_i \delta_i \;; \quad F_{eq} = F_{base} \qquad (2)$$

Calculate and plot the force-displacement curve F_{eq} - δ_{eq} of the equivalent s.d.o.f. system. The evaluation of the dynamic response of the s.d.o.f. system will be made defining a "substitute structure" whose effective stiffness is equal to the secant stiffness $K_{eq,s}$ at a given value of displacement δ_{eq} ..

4) Define the equivalent viscous damping ξ_{eq} (including the effects of hystreretic energy absorption) for the s.d.o.f. susbstitute structure, as a function of the equivalent displacement δ_{eq} , based on the evolution of the damage mechanisms obtained in the pushover analysis, and on energy equivalence principles. Plot the corresponding ξ_{eq} - δ_{eq} curve.

5) Evaluate iteratively the maximum displacement of the s.d.o.f. system consistent with the design elastic displacement spectrum $\delta_{eq,max}$ = SD(T_{eq}; ξ_{eq}), where $T_{eq} = 2\pi\, (M_{eq}/K_{eq,s})^{1/2}$ is the effective period at maximum displacement response.

The sequence of steps from 1 to 5 is based on the results of the pushover analysis carried out with the set of static forces defined at step 1 from an assumed deflected shape. However, in the pushover analysis the ratio of the displacements at each story may vary as a consequence of the nonlinear behaviour of the structure, and the displaced shape corresponding to the value of $\delta_{eq,max}$ calculated at the end of step 5 will differ from what assumed at the beginning of step 1. Depending on the structure, the results of the static analysis may be more or less sensitive to the assumed pattern of static forces, and, in general, the displaced shape will vary continuously as the analysis proceeds in the nonlinear range, differing from a linear or first-mode vibration shape. It may be advisable therefore to repeat the procedure substituting in equation 1 of step 1 the displaced shape obtained at the end of step 5, iterating the whole procedure until a final displaced shape consistent with the assumed force distribution is obtained. However, the need for iteration should not be overemphasized. Given the approximation of a pushover approach, it may be more effective to assume two or three arbitrary displaced shapes consistent with the most probable failure mechanisms (e.g. storey mechanism at the first storey, storey mechanism at the last storey, global overturning) and then follow steps 1 to 5 once for each assumed displaced shape. A range of possible solutions would be obtained, giving a better reference for the assessment. The use of more than one load pattern would be recommended to account for possible higher mode effects [Krawinkler and Seneviratna, 1998].

Equations (2) in step 3 are obtained by simple dynamic and energy equivalence principles and do not need special discussion. Step 4 deserves some comments within this context. The evaluation of a global equivalent viscous damping for a masonry building requires experimental information on the energy dissipation properties of single structural elements (e.g. piers and spandrels). Once the energy dissipation of single elements is defined, it is possible to evaluate the global energy dissipation of the whole structure, and the global equivalent damping. Energy equivalence between the s.d.o.f. substitute structure and the building leads to the following expression for the equivalent damping:

$$\xi_{eq} = \sum_k E_k \xi_k \Big/ \sum_k E_k \qquad (3)$$

where E_k is the elastic strain energy associated to the secant stiffness and ξ_k is the equivalent damping of the k-th structural element. Considering the equivalent frame idealization of the SAM method, the elastic energy of a single beam-column element can be conveniently expressed in terms of moments and chord rotations at the nodes i and j as:

$$E_k = \tfrac{1}{2}(M_{k,i}\theta_{k,i} + M_{k,j}\theta_{k,j}) = E_{k,i} + E_{k,j} \qquad (4)$$

where the work due to axial deformation is neglected. At present, limited experimental information is readily available for URM structural elements in terms of equivalent damping. Herein reference will be made to the work of Magenes and Calvi [1997] who have explicitly evaluated values of equivalent damping for brick masonry piers subjected to in-plane static cyclic loading. On that basis, a first rough approximation can be made to quantify the viscous damping equivalent to hysteretic energy dissipation of a single structural element, depending on the failure mode. In the following application, it has been assumed that piers and spandrels in the linear range are characterized by a constant equivalent damping equal to 5%, and that the value increases to 10 % when one of the shear failure criteria is met. If the element fails in flexure, the equivalent damping associated to hysteretic energy dissipation remains equal to 5 %, but an additional 5 % due to impact and radiation damping is added. The equivalent damping of structural elements will vary therefore in a stepwise fashion. These assumptions aim to give a slightly conservative estimate of the equivalent damping with respect to experimental results. The program SAM has been therefore modified to calculate automatically the equivalent damping of the building according to equations 3 and 4 at every increment of the pushover analysis.

To verify the results that can be obtained by this criterion on a structure, the results of a full scale static cyclic test on a two storey brick masonry building were processed to obtain a reference for the numerical evaluation of the parameters of the substitute structure. The experiment was carried out at the University of Pavia [Magenes et al., 1995], and consisted in a series of displacement cycles of increasing amplitude, applied to the structure keeping a 1:1 ratio among the forces applied at the first and second floor. The longitudinal walls were coupled by flexible floor beams only, so that each longitudinal wall could be analyzed independently as a two-degrees-of-freedom structure. Considering one of the two walls (Figure 9), the experimental response can be evaluated in terms of an equivalent s.d.o.f. structure according to the criteria described above, obtaining the force-displacement diagram of Figure 10. For each cycle it is then possible to calculate the equivalent damping on the basis of the dissipated hysteretic energy and the secant stiffness at peak displacement, obtaining the values reported in Figure 11. The same wall was also analyzed with the SAM method, carrying out a pushover analysis with equal forces at the floor levels, and evaluating the parameters of the s.d.o.f. substitute structure according to the hypotheses described above. Since the test was static, however, impact and radiation damping was not taken into account in the evaluation of ξ_{eq}. A limit chord deformation θ_u = 0.5 % was assumed for piers failing in shear, and the numerical collapse of the structure coincided with the attainment of the limit deformation of the central pier at the ground floor.

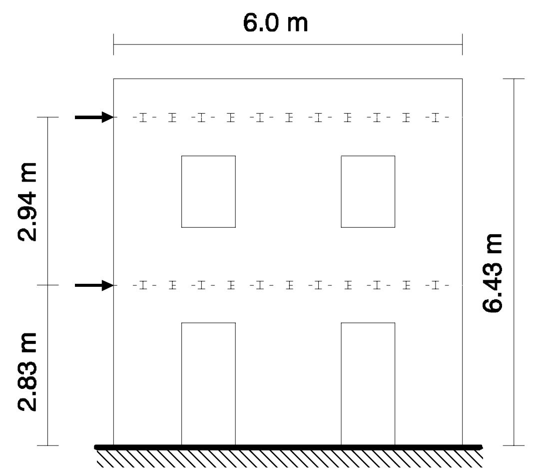

Figure 9. Longitudinal wall of the masonry building subjected to cyclic static testing.

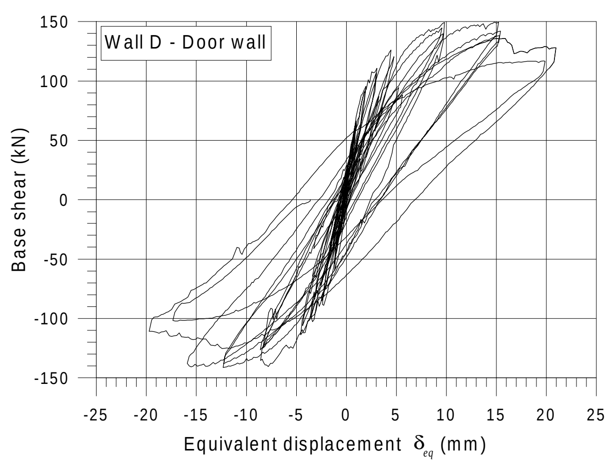

Figure 10. F_{base} - δ_{eq} curve calculated from the experimental response of the wall.

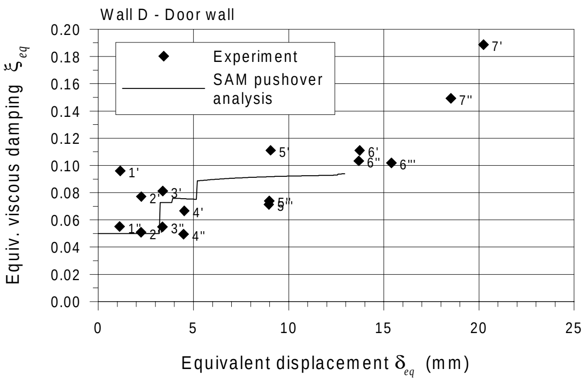

Figure 11. Equivalent viscous damping associated to hysteretic energy dissipation.

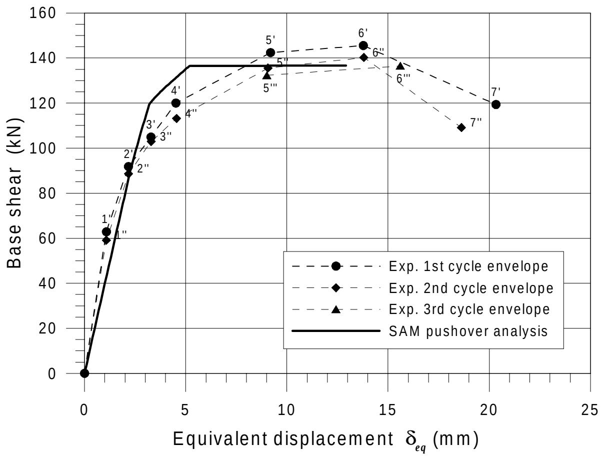

Figure 12. Comparison between experimental envelopes and pushover analysis.

In general, it can be observed from Figures 11 and 12 that the analytical method obtains an acceptable estimate of the nonlinear force-displacement curve and of the equivalent damping. The experimental cycles beyond the displacement of ±15 mm (labeled 7' and 7'') show a significant strength degradation associated to the collapse of a spandrel above one of the doors, which explains the very high experimental value of equivalent damping. Classifying these cycles as "beyond collapse", the comparison between experiment and analysis is meaningful only up to run 6. The "jumps" in the ξ_{eq} numerical curve of fig. 11 correspond to the failure of piers or spandrels in the SAM analysis, according to the assumptions made. A more realistic evolution of the equivalent damping would probably be obtained with a continuous variation of the damping of the elements with increasing angular deformation, although this is not expected to produce significantly different results in the assessment.

A Simple Example of Application

As an example of application of the assessment procedure herein outlined, the ideal two-storey structure represented by the wall of Figure 9 was assessed, assuming a seismic input described by the elastic response spectrum of Eurocode 8 for stiff soil (soil A). The displacement spectrum is obtained from the acceleration spectrum as $SD(T;\xi) = 4\pi^2/T^2\ SA(T;\xi)$. The elastic spectrum defined for a default damping of 5% is scaled multiplying the ordinates by the factor suggested in EC8: $\eta=[7/(2+\xi)]^{1/2}$. A peak ground acceleration of 0.25 g is assumed. The distributed masses are lumped at the storey levels and summed to the masses associated to the floors, giving a total mass of M_1 = 171.1 kN at the first floor and M_2 = 151.9 kN at the second floor. To perform the pushover analysis, a normalized displaced shape corresponding to the first mode of vibration is assumed in step 1: $\{\delta^{(0)}\}^T = \{0.545 ; 1.0\}$ which gives a force distribution $\{F^{(0)}\}^T = F_{base}\{0.38 ; 0.62\}$. The pushover analysis is then carried out (step 2), and the F_{base} - δ_{eq} and ξ_{eq} - δ_{eq} curves of the substitute s.d.o.f. structure are obtained (steps 3 and 4). Step 5 is then carried out by assuming a first value of $\delta_{eq,0}$ equal to the ultimate displacement of the F_{base} - δ_{eq} curve. The corresponding secant stiffness $K_{eq} = F_{eq}/\delta_{eq}$, period T_{eq} and damping ξ_{eq} are evaluated and the displacement spectrum is entered to obtain a new value of displacement $\delta_{eq,1}$. At this first iteration, by checking if $\delta_{eq,1} \leq \delta_{eq,0}$ it is already possible to verify if the ultimate displacement $\delta_{eq,ult}$ of the structure will not be exceeded. If this is verified, with a trial- and-error procedure it is possible to converge to a final value of displacement such that $\delta_{eq,n+1} \cong \delta_{eq,n}$ within a specified tolerance. In the case considered this results in $\delta_{eq,max}$ =12.1 mm, compared to an ultimate displacement $\delta_{eq,ult}$ = 13.6 mm. At this point, the displaced shape $\{\delta^{(1)}\}$ corresponding to the "design" displacement of 12.1 mm can be checked and compared with the initial assumed displaced shape $\{\delta^{(0)}\}$. In this case, after normalization, the displaced shape $\{\delta^{(1)}\}^T = \{0.804 ; 1.0\}$ is obtained, which shows a storey mechanism at the first storey. As it can be observed, the calculated inelastic response may lead to a displaced shape which is rather different from an elastic first mode shape. If now a new distribution of seismic forces $\{F^{(1)}\}$ is calculated from $\{\delta^{(1)}\}$, obtaining $\{F^{(1)}\}^T = F_{base}\{0.475 ; 0.525\}$, steps 1 to 5 can be repeated, defining new F_{base} - δ_{eq} and ξ_{eq} - δ_{eq} curves. It may be worth to notice that now the force distribution is approaching a constant force distribution, which is consistent with the storey mechanism obtained. At the end of this second global iteration, the following results are obtained: $\delta_{eq,max}$ = 5.5 mm, $\{\delta^{(2)}\}^T = \{0.668 ; 1.0\}$. A further iteration yields $\delta_{eq,max}$ = 6.0 mm, $\{\delta^{(3)}\}^T = \{0.665 ; 1.0\}$. The similarity of the values calculated in the last two iterations suggest that the value $\delta_{eq,max}$ = 6.0 mm can be considered an acceptable estimate of the maximum response of the structure.

CONCLUSIONS

From what presented in this paper, it appears that the recent trends which are being followed in seismic design and assessment of structural types such as reinforced concrete and steel structures can be pursued also for masonry buildings.

On one hand, the proposed model for nonlinear static analysis has so far produced satisfactory results. Still, further comparisons with other methods of analysis on different structural configurations are needed and are presently carried out as a part of the ongoing research. The features that make the SAM method attractive for the applications are mainly the low computational burden and a good versatility. This second feature allows the engineer to select among a range of possible solutions and hypotheses, to compare the most realistic with the most conservative, allowing to draw sounder conclusions for the assessment, especially when the knowledge of the existing structural system is incomplete, as can be the case for historical buildings.

On the other hand, it is clear that a satisfactory model for monotonic analysis is not sufficient for a reliable prediction the dynamic response under seismic excitation. The proposed assessment procedure based on the substitute structure concept appears to be a step forward with respect to current codified practices, however its effective capability of predicting correctly the maximum dynamic response needs further verification by comparison with dynamic analyses and with experiments. In fact, unreinforced masonry structures presents specific features (history-dependent degradation of stiffness and strength under cyclic actions, sensitivity to the duration, frequency and energy content of the seismic input) which must be carefully considered for the definition of a reliable assessment procedure. The future research will therefore be dedicated to the study of such aspects.

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